APPRAISAL REPORT

CHICOPEE, MASSACHUSETTS

CONNECTICUT RIVER

LOCAL FLOOD PROTECTION

MARCH 1985

US ARMY CORPS
OF ENGINEERS
NEW ENGLAND DIVISION

APPRAISAL REPORT
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TABLE OF CONTENTS

I. INTRODUCTION

A. Authority

The city of Chicopee, Hampden County, Massachusetts is located on the Connecticut River at its confluence with the tributary Chicopee River. The mouth of the Connecticut is about 80 miles downstream. There are approximately 1100 acres being protected, including many residences and industries.

The Chicopee Local Protection Project (LPP) is a unit in the comprehensive flood protection plan for the Connecticut River Basin authorized by the 1936 Flood Control Act and modified by the 1938 Flood Control Act.

EC 11-2-147 provides direction to review the adequacy of completed LPP's which were specifically authorized by Congress. Development in watershed areas and new information on basin hydrology since the project's construction may warrant an updated analysis of the degree of protection being realized. The objective is to determine whether it is advisable to modify the structure due to changes either in the area being protected or to make changes to the project to improve its viability, safety, and reliability.

B. Purpose and Scope.

The purpose of this investigation is to assess and document the adequacy of modifying the existing LPP on the Connecticut River through Chicopee, Massachusetts, and determine if modifications are advisable and warrant further Federal study.

The scope of this particular report is of a reconnaissance nature. The objectives are:

- Compile existing information
- Initiate public involvement
- Establish the need for modification
- Identify modification opportunities
- Determine preliminary feasibility of modifications
- Recommend future course(s) of action

The study process is divided into two phases - reconnaissance and feasibility. In reconnaissance, modifications to the project are screened from the standpoints of economic, environmental, and engineering integrity and safety considerations. The detail used is strictly at the level of

initial appraisal. Items of local cooperation, both past and future, are addressed when an affirmative action is recommended.

If warranted, the feasibility phase would detail the actual modification alternatives and recommend a particular course of action. The recommendation would be based on a comparison of each alternatives expected accomplishments.

C. Public Coordination

The city of Chicopee was notified by letter, dated 16 May 84, of the New England Division's (NED) initiation of study efforts to review the existing LPP for the advisability of possible modification.

On 9 August 1984, personnel from NED visited the project and protected area. Meetings were held with the city's Planning Director and Engineer to discuss the investigation and obtain their views. Both cited local funding as their main restriction toward keeping the project in satisfactory condition.

D. Other Studies

1. The most recent semi-annual inspection was conducted 30 October 84. The project is in unsatisfactory condition. Deficient items of significance noted include:

. Pumping stations in urgent need of structual rehabilitation and mechanical replacements.

. Vegetation growth along dikes needs to be removed.

2. In accordance with revised operating procedures established in February 1984, a new Operation and Maintenance Manual is being finalized and will be distributed to local officials shortly.

Preceding this, NED's Water Control Branch completed a report in February 1983 which included a hydrologic review, update, and analysis of interior drainage facilities at the Chicopee LPP. The purpose of the review was to provide a hydrologic assessment of present interior drainage conditions relative to pumping station needs to aid in the planning and prioritizing of plant replacements and improvements.

3. The Federal Emergency Management Agency's Flood Insurance Study for the city of Chicopee became effective March 1978.

4. The New England River Basins Commission prepared a unified program for flood plain management in the Connecticut River watershed in 1976. This "River's Reach" examined many alternatives and approaches to solving the flood problem. One of these was raising the existing LPP's to provide more protection.

III EXISTING CONDITIONS

A. Project History and Description

1. Construction

The existing project was built as a result of the disastrous Connecticut River flood of March 1936. It was built in sections, starting in October 1936 and ending in 1941. The LPP consists of a system of earth dikes at minimum top elevation 72.4 ft. NGVD and concrete floodwalls at minimum top elevation 70.6 ft. NGVD extending approximately 25,700 feet along the left (east) bank of the Connecticut River and both banks of the Chicopee River. The upstream end of the Springfield, Massachusetts, dike and its pumping station is also located within the city of Chicopee, Massachusetts. Although this segment is hydrologically separate, it is considered part of the Chicopee LPP. A map of the project area is shown on Plate 1.

The Chicopee project includes six pumping stations for discharging interior drainage during high river stage, and three stoplog features through the dike permitting passage of rail and vehicular traffic during non-flood periods. As most of Chicopee is serviced by a combined sanitary and storm sewer system, the pumping stations were designed to discharge both flows during high river stage.

The project's first cost was $1.7 million in 1941. This included items of local cooperation (lands, easements, rights-of-way, etc) amounting to $250 thousand. By comparision, this same construction cost in today's dollars would be over $27 million!

The Chicopee Falls LPP, also shown on plate 1, consists of a system of earth dikes (3,600 feet) and concrete floodwalls (1,400 feet) extending along the left bank of the Chicopee River, two pumping stations, two pressure conduits and water intake lines. These measures were constructed in the aftermath of the Chicopee River flood of August 1955. Construction was started in October 1963 and completed in July 1965.

The first cost of the Chicopee Falls LPP was $2.6 million in 1965. Items of local cooperation were $400 thousand. This construction would be equivalent to about $11 million at current price levels.

(a) Plainfield Pumping Station

The Springfield LPP along the east bank of the Connecticut River extends northward a short distance into the city of Chicopee. The protective works within Chicopee intercept runoff from about 300 acres

within the political limits of Chicopee. The interior drainage from 90 percent of that area is served by a 72-inch pressure storm drain discharging to the Connecticut River just downstream of the Plainfield pumping station. Drainage from the remaining 10 percent (30 acres) is normally discharged by gravity, but the Plainfield pumping station was built to discharge this runoff during high river stages.

The 30 acres served by the station is a very flat industrial area with limited storm drainage facilities. The Plainfield station was built with provisions for the future connection of twin 36-inch diameter storm drains. These have not been installed and no plans exist for their future installation. Presently the only drainage entering the station is from 8 and 12-inch toe drains located along the land side of the line of protection. Estimated capacity of these toe drains is in the order of 8 cfs.

(b) Dwight Street Pumping Station

There were four tailraces servicing the water power canal that drains into the Chicopee River. The Dwight Street Pumping Station is located at the outlet, through the line of protection, of tailrace 3 as depicted on Plate 2. The pumping station was designed to discharge interior drainage comprised of:

(1) Gate leakage at tailraces 1, 2, and 3 (35 cfs)
(2) Seepage through the water supply canal embankment (20 cfs)
(3) Seepage through the dike and under the floodwall (4 cfs)
(4) Leakage from tailrace 4's pressure conduit (10 cfs)
(5) Industrial water (15 cfs)
(6) Interior storm runoff (25 cfs).

Tailrace canal 3, which originally provided an interior ponding area, has been replaced by a pipe conduit and almost completely filled in for use as a parking lot. The industry located east of Davitt Memorial Bridge is no longer in business and the buildings are presently vacant and in dis-repair. It is presently planned that the buildings will be converted to housing. With tailrace canals 1 and 2 no longer in use, their gates are closed.

The Dwight station has twin 9 feet x 9 feet gravity sluice gates which are both presently inoperable. One is in the closed position and the other almost closed. Two of the three pumps are presently operational. In addition, the exterior brickwork requires extensive repair and waterproofing.

(c) Bertha Avenue Pumping Station

The Bertha Avenue station receives drainage from a bluff area east of the Connecticut River flood plain. Drainage from the bluff passes west beneath the new highway route 391 and then south through a low flat area along the B&M railroad. It is noted that storm drainage from route 391

does not reach the Bertha station. The highway has its own extensive drainage system discharging directly to the Chicopee and Connecticut Rivers. The Bertha station was originally designed for a drainage area of 335 acres, however, the present drainage area now extends further north along the B&M railroad and is more nearly 400 acres. The station was designed with about 15 acre-feet of ponding capacity adjacent to the station. Much of this storage however has been lost with the construction of the highway. The remaining storage is that within a highway interchange loop just upstream from the station. The storm drain capacity between this upstream interchange storage and the pumping station is that provided by twin 36-inch diameter culverts.

(d) Paderewski Pumping Station

The Paderewski pumping station was built to provide interior drainage, under high river stage, for about 350 acres of very flat residential area lying between the Massachusetts Turnpike north to Chicopee Street. The original station was designed for a drainage area of 260 acres, however, the watershed served now is more nearly 350 acres. The area has a combined sanitary-storm sewer system culminating in a 60-inch diameter drain at the pumping station. A recently constructed sanitary interceptor sewer system diverts normal sanitary flows to the station, but the diversion capacity from the Paderewski watershed is only 1 to 2 cfs. Therefore, the station provides little relief to the storm drainage requirements from the area.

The Paderewski station was originally equipped with two - 30-inch propeller pumps and a 16-inch variable speed, electrically driven, volute pump with provisions for the addition of another 30-inch propeller pump when expected development in the area warranted. One of the 30-inch propeller pumps was recently replaced. The structure itself is in a state of disrepair. The roof needs replacement and the exterior requires extensive work.

(e) Jones Ferry Pumping Station

The Jones Ferry pumping station located near the end of McKinstry Street was built to discharge the flows of a 72-inch diameter combined sewer outletting to the Connecticut River. In the original design, it was assumed the sewer served an interior watershed area of about 840 acres. This area was the very flat Connecticut flood plain area west of the B&M railroad and a high bluff area lying east of the railroad. It was determined that a 48-inch diameter pressure conduit intercepts much of the drainage from the area east of the railroad and discharges it to the Connecticut River just south of the Jones Ferry pumping station. This pressure conduit reduces the contributing watershed to the pumping station from the original 840 acres to more nearly 630 acres. The only drainage from east of the railroad to the pumping station is now from one 30-inch drain, with an estimated capacity of 20 cfs. The present drainage area is

very flat, mostly residential, with some industrial and commercial development. Also, the area is almost completely storm sewered. Principal sewers feeding the trunkline are two 36, one 48 and one 42-inch diameter drains. The conveyance capacity of the existing drain to the pumping station is about 200 cfs.

A diverter has been installed on the trunkline sewer near the station for sending "dry weather" flows to a treatment plant. However, diversion capacity from the watershed is limited to about 5 cfs and has little effect on the peak rate of storm runoff to the Jones Ferry station.

The Jones Ferry station was originally equipped with three 42-inch propeller pumps and one 16-inch variable speed volute pump with provisions for the future addition of a fourth 42-inch pump. Two of the large pumps are currently inoperable. Like most of the other stations, the structure itself is in urgent need of attention. The exterior brickwork and windows require extensive repairs.

(f) Call Street Pumping Station

The Call Street pumping station, the most northerly station in Chicopee, serves to discharge the flows of a 60-inch combined sanitary storm sewer to the Connecticut River during periods of high river stage. The station was originally designed to serve an eventual maximum watershed area of 740 acres. The present, and probable future, total drainage area is considerably less. About one-half the original drainage area was the flat flood plain area west of the B&M railroad and the other half the steeper bluff area east of the railroad, generally divided by Gratton Street. The surface runoff from the bluff area south of Gratton Street drains west beneath highway 391, and then south along the B&M track, eventually reaching the Bertha Avenue pumping station. Also, the surface runoff in the draw north of Gratton Street enters a 48-inch diameter culvert inlet at the B&M railroad, and indications are, based on discussions with city personnel and a study of drainage maps of the area, that this 48-inch drain flows due west outletting to the Connecticut River upstream of the Call Street station. The present Call Street station drainage area is more nearly 230 acres, mostly of very flat residentially and commercially developed flood plain area west of the railroad plus a strip along Gratton Street east of the railroad.

It would be expected that during intense rainfalls some flows would bypass the inlet to the 48-inch drain north of Gratton Street and enter the flat flood plain, but would likely pond regardless of pumping capacity due to the limited conveyance capacity to the station. An important consideration in the hydrologic evaluation of Call Street is that the present storm drain capacity to the station is estimated to be about 110 cfs, and there are no plans by the city for enlarging this capacity.

Only two of the station's large pumps are operable. In addition, the exterior brickwork and windows need major repair. The roof also requires attention. If these items are not addressed, the structural integrity and mechanical equipment inside may be threatened.

(g) Main Street Pumping Station

Interior drainage from approximately 150 acres at the Chicopee Falls project is intercepted and discharged to the river by pressure conduits. Drainage from a remaining 38 acres is provided by two pumping stations, during high river stage, Main and Oak Street. The Main Street pumping station is located adjacent to the line of protection (floodwall) approximately 2,000 feet downstream from Deady Memorial Bridge and discharges drainage from about 20.5 acres of the industrial area at the former Savage Arms and Chicopee Manufacturing Corporations. Drainage to the station is by 30 and 18-inch diameter storm drains with a total combined capacity of 30 to 40 cfs.

This station is probably in the best condition of all of them. The most recent inspection found that only the sump intake gate does not close. The Chicopee Falls general plan, Plate 3, depicts both the Main Street station and the Oak station described below.

(h) Oak Street Pumping Station

The Oak Street station, with a drainage of almost 18 acres, serves the most downstream portion of the interior drainage area. The area is the large industrial complex of the former US Rubber Company. Unfortunately the industry at the site has closed down and the extensive building complex is rapidly deteriorating. The station is fed by 36 and 30-inch diameter drains with an estimated combined capacity in the order of 50-60 cfs. The Oak Street station has a design capacity of 46 cfs, 20 of which is allotted to process water and 26 for interior storm runoff.

All the pumps within the station are operable. However, the sump intake gate, as in the Main Street station, does not close. This situation should be corrected as soon as possible.

(i) Stoplog Structures

The three railroad and vehicular openings included in the Chicopee protection works are:

Structure #1 - Railroad opening northwest of Depot Street, 8 feet high x 19 feet wide, sill elevation at 64.9 feet NGVD.

Structure #2 - Vehicular traffic opening on Depot Street 5 feet high x 55 feet wide, sill elevation at 65.4 feet NGVD.

Structure #3 - Railroad opening west of Bertha Avenue, 5.5 feet high x 31 feet wide, sill elevation at 67.5 feet NGVD.

The rate of rise of the river during a storm event is expected to range up to 0.8 foot per hour during the period required to effect closure. The O&M manual indicates that the openings are to be closed when the Connecticut River has risen to the following elevations at the I-90 gage:

63 feet NGVD - Stoplog Structure No. 1
64 feet NGVD - Stoplog Structure No. 2
66 feet NGVD - Stoplog Structure No. 3

2. MODIFICATIONS

Within the past few years a sewage treatment system has been built in Chicopee in which "dry weather" flows from the combined sewer system are intercepted near outlets to the river and conveyed to a central point for treatment and discharge to the river. It was determined that the diversion capacity of this treatment system is equivalent to a watershed runoff rate of only about 0.14 inches per day (3-4 cfs/square mile). During intense rainfall, therefore, the existing storm drainage discharge capacity is only minimally supplemented.

The Depot Street stoplog structures were originally designed to prevent Connecticut River flood levels from backing up through the canal, overtopping Depot Street and overflowing into the protected area behind the dike. However, when I-90 was built, reinforced concrete walls along both sides of the Depot Street bridge crossing over the canal replaced the dike sections. The top of these walls are at 70.5^{+}_{-} feet NGVD, and would prevent overtopping from the design flood.

3. Damages Prevented

The Chicopee LPP has prevented nearly $10.5 million in flood damage since its construction. Almost 43 percent of this was during FY 84 alone. Chicopee Falls LPP, similarily has avoided just over $800,000 in flood losses to date - approximately 60 percent of which during FY 84.

The two most recent instances in which the Chicopee and Chicopee Falls LPP's prevented flood damage were the events of April and May/June 1984. The damages prevented (benefits) are allocated between the LPP's and the existing system of reservoirs. Damages prevented are determined by comparison of the actual observed flow, with the existing upstream reservoir system in place, to the calculated natural flow that would have occurred without the reservoirs.

For the May/June 1984 event the natural flow on the Connecticut River thru Chicopee would have been 225,900 cfs. Flood losses associated with this would have amounted to $11,332,000. However, the actual observed flow was only 189,200 cfs which meant a reduction in potential damages of $6,949,000 by the reservoir system. Since flood losses in the area protected by the Chicopee project would begin at a flow of 90,000 cfs, the LLP was credited with preventing the remaining $4,383,000 in flood damages from that event alone. The following table displays damages prevented by the Chicopee LPP for recent selected events.

TABLE 1
DAMAGES PREVENTED IN THE CHICOPEE LPP AREA

Event	Start of Damage (cfs)	OBSERVED CONDITIONS		COMPUTED NATURAL CONDITIONS		DAMAGES PREVENTED	
		cfs	Damages	cfs	Damages	Dams	LPP
May/Jun 1984	90,000	189,200	$4,383,000	225,900	$11,332,000	$6,949,000	$4,383,000
April 1984	90,000	99,600	$63,900	114,000	$81,100	$17,200	$63,900
Feb/Mar 1981	90,000	96,500	$41,800	106,800	$59,200	$17,400	$41,800

In the Chicopee Falls area a calculated natural flow was not determined for the May/June 1984 event. The actual observed flow was 30,000 cfs reflecting the effects of the upstream Barre Falls and Conant Brook dams. Since damages would have begun at a flow of 20,000 cfs in this area, the LPP was credited with preventing potential damages of $510,000 for that event.

The Chicopee LPP has prevented nearly $10.5 million in potential flood losses since its construction. Would be 1984 damages make up about 42 percent of that total. The Chicopee Falls LPP, similarily, has avoided just over $800,000 in potential flood losses to date, 60 percent of which were in 1984.

4. Level of Protection

(a) Chicopee LPP

This project provides protection against flood stages on the Connecticut River and concurrent backwater flooding on the lower Chicopee River, designed against a Connecticut River flood discharge of 312,000 cfs. A Standard Project Flood (SPF) along the Connecticut River would overlap the LPP. However, the project can protect against an event having an annual chance of occurrence approaching 0.1 percent. The dikes and floodwalls provide 3 feet and 1 foot of freeboard, respectively, above the record March 1936 flood level, or about 8 feet and 6 feet above the recurring March 1936 flood as modified by the Corps existing system of flood control reservoirs.

(b) Chicopee Falls LPP

Protection is provided against a Chicopee River discharge of 70,000 cfs, which is the standard project flood modified by the Barre Falls and Conant Brook reservoirs. The design discharge exceeds the 42,500 cfs experienced during the record flood of September 1938 at the Indian Orchard gage. The top of the dikes and floodwalls provide 3 feet of freeboard above the design discharge.

5. Recent Inspections

The most glaring and immediate problem with the existing measures comprising the Chicopee LPP is the condition of the pumping stations. Although many of the pump engines were operated satisfactorily, numerous mechanical deficiencies were noted. These must be corrected to ensure proper operation during a flood condition. The last semi-annual inspection was conducted on 30 October 1984. The project, at that time, was in unsatisfactory condition.

The Call Street, Jones-Ferry, Paderewski Street, Bertha Avenue and Dwight Street pumping stations are in need of extensive structural rehabilitation. Exterior concrete and masonry work, window panes and screens, and roofs all need immediate attention to prevent future damage to any mechanical equipment inside from vandalism or the weather.

Finally, the inspection report identified the need to control vegetation on the dike system. If left untreated, the roots could threaten the structures' integrity. However, it was observed at a follow-up meeting with city personnel on 9 August 1984 that initial efforts were being undertaken to improve this situation.

B. Project Area

1. Description

Many of the homes offered protection currently have been built since the project's construction. However, the intensity of industrial use within the protected area has definitely declined in that same period. In fact some of the original buildings have been left vacant or have been removed. This area is not the center of economic activity it once was.

2. Hydrology and Hydraulics

The average annual precipitation over the basin is approximately 43 inches, and varies from about 36 inches along the Conncticut River valley to more than 60 inches in the White and Green Mountains. The annual precipitation in the Chicopee area is 44 inches.

Precipitation in the northern one-half of the basin during the winter months is practically all in the form of snow; in the southern areas alternate periods of snow and rain can be expected. The snowfall varies

alternate periods of snow and rain can be expected. The snowfall varies from an average of less than 40 inches annually at the lower elevations in Connecticut to well over 100 inches in the northern and mountainous areas of the basin.

Early spring thaws usually diminish the snow cover in the lower elevations of Massachusetts and Connecticut before melting takes place in the higher elevations or northern areas of New Hampshire and Vermont. Water content of the snow in the mountains often reaches 6 to 10 inches. The water content in the snowpack usually reaches a maximum about the middle of March.

The average annual runoff for the basin is about 23 inches or slightly over one-half of the average annual precipitation. The annual runoff follows a pattern somewhat similar to the annual precipitaion in that it varies from about 17 inches in the areas of lowest elevation in the main river valley north of the Massachusetts State line, to more than 40 inches in the highest elevations of the White and Green Mountains. About 50 percent of the annual runoff in the central and northern portions of the watershed occurs in the spring months of March, April and May. Runoff in the lower basin during the same months, as a result of less snow accumulation, is about 40 percent of the annual.

Flooding may occur in the basin during any season of the year, resulting from excessive rainfall, snowmelt or a combination of both. In the spring months, flooding is usually associated with snowmelt throughout the basin, resulting in prolonged high stages on the main stem and a large volume of runoff. The floods of March 1936, April 1960, and January 1949 were of this type. The fall floods of November 1927 and September 1938 and the summer floods of August 1955 resulted from intense rainfall. Following is a summary of recent flood data for nearby gaging stations on the Connecticut and Chicopee Rivers. Plate 4 depicts Connecticut and Chicopee River's discharge and stage-frequency relationships.

TABLE 2
FLOOD DATA

Connecticut River at Montague City, Mass. (DA = 7,865 sq. mi.)			Chicopee River at Indian Orchard, Mass. (DA = 688 sq. mi.)		
Date	Peak Discharge (cfs)	River Depth (ft)	Date	Peak Discharge (cfs)	River Depth (ft)
Mar 1936	236,000	49.2	Sep 1938	45,200	-
Sep 1938	195,000	44.7	Aug 1955	40,500	22.1
Nov 1927	179,000	-	Mar 1936	20,400	-
Apr 1960	142,000	38.1	Mar 1980	10,500	12.4
Jan 1949	139,000	37.8			

III. FUTURE CONDITIONS

A. Land Use

1. Community Plans

The protected area is made up of primarily residential neighborhoods and some industry along the Connecticut's shore. The construction of the project was prior to a good deal of the homes presently there. The area is fully developed, with little room for future growth.

Discussions with Mr. Frank Rueli, the City Engineer, and Mr. Mark Webb, a City Planner found that Chicopee does not have any specific strategy for the protected area. The zoning is mixed and is a strong indicator of future use.

Many of the Old Mill buildings that were the source of project benefits are in a state of disrepair or gone all together. Although the city would like to revitalize the area, there are no plans currently to that end.

2. Economics

At the time of the Chicopee project's construction, potential losses were distributed among land use categories in the following percentages: industrial (74%), urban (19%), highway (5%), rural (17%) and railroad (1%). The protected area was reviewed for property changes in 1948 and again in 1954. The 1954 land use percentages identified a trend that is expected to continue. The percentage of damages to industrial properties fell over the 1938-1954 period from 74 percent to 65 percent, while the urban category (residential and commercial) grew from 19 to 29 percent. The 1954 study identified a complex of 63 buildings containing 226 apartments built in 1951 and 81 single family homes, then under construction. Field inspection in 1984 indicates that new single family and duplex homes were built in the very recent past in the southern one-third of the protected area.

Based on existing flood loss data, a recurrence of a flood of the magnitude of the March 1936 event in the Chicopee local protection area, with no LPP or upstream reservoir system would result in $43,000,000 in flood losses. In the area protected by Chicopee Falls LPP, a recurrence of the flood of the magnitude of August 1955 event, with no upstream reservoirs or LPP in place, would result in $7,400,000 in flood losses.

B. Project Integrity

The existing LPP's have performed the intended purpose over their lives to date. With appropriate operation and maintenance the project's should be able to continue doing this. However, as the inspection reports indicate, the project is currently in an unsatisfactory condition.

The system of pump stations need immediate attention. Without their proper function during storm events, ponding within the protected area could be extensive and cause flood damage. This would defeat the projects' purpose.

IV. CURRENT PLANNING AND DESIGN CRITERIA

A. Freeboard

1. Requirements

There are no specified criteria with regard to the design level of protection for flood damage reduction projects. Each project should be complete within itself and provide the maximum net benefits, unless there is overwhelming justification to deviate. In urban areas the Standard Project Flood is a design goal since potential overtopping or failure could be catastrophic.

Engineering regulations call for freeboard allowances above design grade of 2 feet for concrete walls and 3 feet for dike or levee systems. With the existing system of reservoirs in the Connecticut River Basin, the Chicopee and Chicopee Falls LPP's conform to this criteria.

2. Economics

Current planning guidance allows for taking credit for expected benefits within the freeboard range. In the case of the Chicopee LPP, this is not applicable since the elevation of the mid-point of the current freeboard is below the elevation where benefits were credited to when the project was originally planned. Benefits for the lower half of the current freeboard of the Chicopee Falls LPP would be insignificant due to the high level of protection already provided.

EM 1120-2-104 outlines the procedure regarding benefits for advance replacement of existing projects. A credit can be taken for extending the life of a project and realizing benefits beyond which the project would have continued to function.

Since the Chicopee the LPP is 43 years old, and near the end of its economic life, any modification that extends its physical life may take advance replacement benefits. However, an engineering analysis of the structure's stability and integrity would have to be accomplished to determine just how much longer the LPP can perform its intended purpose since advance replacement benefits can only be attributed for the period of time after that. This study does not address this issue.

V. MODIFICATION OPPORTUNITIES

A. Level of Protection

In 1976, the New England River Basins Commission reported in their River's Reach on the feasibility of raising the Chicopee LPP. Protection up to the SPF was examined involving increasing concrete flood wall heights 4.7 feet and earth dikes 4.9 feet. This was estimated to cost almost $16 million at 1974 price levels, with a benefit to cost ratio of only 0.1 to 1.0. Today it would cost over $32 million.

Although the city of Chicopee and the Commonwealth of Massachusetts expressed an interest in feasibility studies back in 1976, the costs clearly outweigh the potential benefits and preclude modification at this time. A recent inspection of the protected area did not find any change in this potential benefit to cost relationship.

B. Protected Area

Inspection of the areas downstream and upstream of the existing projects indicate extension of the protection is not needed at this time.

C. Project Features

1. General

As discussed earlier, the pumping stations need urgent rehabiliative attention. The assessment of the relative adequacy of a station cannot be entirely analytical, but should be a combination of both quantitative and subjective analyses. The sizing of a station must be weighed against the flood risk based on both frequency and magnitude of potential damages. When the flood damage potential involves loss of life or extensive property damage, a stringent design criteria of the 1 to 2 percent chance of annual occurence, or a storm event even more rare, is generally employed. Some hydrologic factors to be considered in assessing pumping station requirements are:

- Frequency and duration of high river stages that require pumping.
- Likely coincidence of interior rainfall-runoff and high river stage.
- Interior watershed size and runoff potential.
- The interior flood damage potential in both human life and property.

A guide chart was developed in which design runoff criteria is related to flood damage potential, frequency of high river stage and rainfall-river stage coincidence is shown on Plate 5.

The frequency and duration of a river's flood stages preventing drainage by gravity is an indicator of the need for pumping. The greater the frequency and duration of high stage, the greater the chance of intense interior rainfall occurring during that high river stage. This is a measure of risk based on probability of occurrence alone, and is an important consideration in combination with other factors such as the magnitude of potential damage.

Stage-frequency curves for the Connecticut and Chicopee Rivers are shown on Plate 4. Activation of the six pumping stations at the Chicopee LPP are a function of Connecticut River stages, and the two stations at the Chicopee Falls project area a function of Chicopee River stages. The approximate upper limit river stage and frequency for pump activation at each of the stations is indicated on the curves. The stage-frequency curves were developed using peak discharge-frequencies, as modified by upstream flood control reservoirs. Discharge-frequency and stage-discharge ratings are also shown on Plate 4. The analysis indicated that the frequency of required pump activation at the different stations ranged from about an annual event at some stations to as rare as an event with a 2 percent annual chance of occurrence at the Plainfield station.

With protection projects on small flashy rivers it is quite probable that interior rainfall will occur during high river stage, both resulting from the same storm system. However, on larger rivers, like the Connecticut, with increasing times of concentration, interior rainfall during high river stage would be less probable and more likely the result of a secondary storm system rather than that associated with the initial runoff event. Considering the likely coincidence of the two events is again a relative measure of risk based on "probabilities" rather than "possibilities" of occurrence.

Rainfall amounts in the region were studied for periods when the Connecticut River was approaching or exceeding minimum stages for pump activation. Though the Connecticut and Chicopee Rivers are high a small percentage of the time, the analysis indicated a relatively high coincidence of interior rainfall during high river stages. This was most notable during the greater flood events, particularly the major floods of August 1955 and September 1938. The analysis was made for a 23-year period and the highest experienced 1 and 3 hour rainfall amounts during high river stage were listed in order of magnitude and assigned "Weibull" annual frequency plotting positions. The developed rainfall frequencies are shown on Plate 6 and are compared with all season rainfall frequencies from US Weather Bureau T.P. #40. This comparison indicated that the 40 percent chance all season rainfall had about a 2 to 4 percent annual chance of occurrence during high river stage and the 10 percent all season rainfall about a 1 to 2 percent chance of occurrence during high river stage.

It is noted that in August 1955, a 5 to 10 percent chance all season 3-hour rainfall was experienced during high river stage. The pumping stations operated satisfactorily, but widespread interior flooding did occur, however due to insufficient storm drain capacity.

The primary factor in determining interior runoff potential is the size and character (topography and development) of the interior watersheds. In addition, as previously discussed, the peak rate of runoff to a pumping station from a relatively flat urbanized area can be highly affected and limited by the design capacity of storm drainage systems in the area. The projects' pumping stations were originally designed using runoff rates based on 10 percent annual chance of occurrence rainfall and reduced using a "Relative Protection Factor" adjustment.

The adopted upper and lower limit curves of runoff rate versus frequency are shown on Plate 6. The lower curve is considered typical of an average undeveloped watershed in the area, and the upper curve represents peak runoff frequencies for a steeper more flashy watershed. The curves indicate peak discharge frequencies for a one square mile watershed. Peak discharges generally vary with watershed size by a ratio of drainage are to the 0.7 exponential power. The peak discharge frequency from a given size watershed is equal to the peak flow for a one square mile drainage area, of similar hydrologic character, multiplied by the given drainage area in square miles to the 0.7 exponential power.

Runoff volume frequencies for durations up to 4 days were similarly developed from high flow duration studies of long term streamflow records in the general region. The approximate runoff volume-frequency relations are also illustrated on Plate 6.

2. Assessment

The interior flood potential in the Plainfield area is rated "low to medium". A major storm runoff of 3 to 4 inches, with discharge preempted, would result in flooding in the 2 to 3 foot maximum depth category. The frequency of high river stage requiring pumping, about 2 percent chance of annual occurrence is rated "low". Under present levels of development in the area and the storm drainage inlet restrictions, pumping capacity greater than the existing 8 cfs is considered unwarranted. Present inflow is limited to the capacity of the 8 and 12-inch diameter toe drains with an estimated combined capacity of about 8 cfs.

The interior flood potential at the Dwight station, from interior storm runoff, is also considered "low to medium". A 4-inch rainfall runoff without discharge would result in ponding of 2 to 3 feet in parking lots and industrial building basements. The frequency of high river stage requiring pumping is "medium". The maximum pump activation stage of 58 feet NGVD has about a 10 percent annual chance of occurrence. Based on potential damages and stage frequency for pump activation, the pumping capacity of 25 cfs allotted to interior storm runoff, estimated to be at

least a 20 percent all season frequency runoff, is considered both adequate and appropriate. Since tailraces 1 and 2 are no longer used, it is recommended that the gates be both closed and sealed to prevent leakage. This would theoretically reduce design pumping capacity requirements by 15 cfs - that allotted to leakage. Without this leakage and the 15 cfs allotted to industrial water supply, originally designed pumping requirements would be reduced from 110 to 80 cfs and might be provided by two rather than three pumps.

Complete rehabilitation of the Dwight station gravity sluice gates is of highest priority. The present twin 9 foot square gates were originally designed to pass maximum tailrace channel 3 flows of about 350 cfs, with minimum head loss. Since tailrace channel 3 has been mostly filled and replaced by a storm drain, it follows that the required sluicing need has been greatly reduced. Assuming the drain replacing tailrace 3 isn't more than 48 inches in diameter, it is concluded that a gravity sluicing capacity of about 200 cfs would be adequate. This would provide a 1 percent chance of annual occurrence storm runoff gravity capacity of 100 cfs (4" R.O./hr), plus another 100 cfs for leakage and seepage. The required gate opening would only be in the order of 20 to 25 square feet. Twin 3-foot wide by 4-foot high gates, or the equivalent, could replace the present twin 9-foot square gates.

The flood damage potential in the Bertha Avenue interior area is considered "low to medium". Flooding in the event of excess runoff would be in the 2 to 4 foot category, generally concentrated in the low area along the west side of the B&M railroad. Flooding would occur quite extensively throughout the industrial developments present in the area and it is expected, with the construction of route 391, that there will be expanded development in the future here also. The frequency of high river stage requiring pump activation is "medium".

The total installed pumping capacity of the two pumps at the station is about 70 cfs. With the increased drainage area, storm drain improvements that have been made, and the loss of ponding area, this total capacity is likely not more than a 20 to 50 percent annual chance all season runoff rate and could be even less with continued storm drain improvements. The capacity of the twin 36-inch culverts leading to the station are in excess of 100 cfs.

It is concluded that the Bertha station does <u>not</u> have excessive capacity for present or expected future drainage conditions in the watershed. The station should be properly maintained with both pumps operational and any replacements should be of no less capacity. It is also hydrologically important that the remaining ponding capacity in the highway interchange immediately upstream be retained. The gravity discharge capacity at the station is also limited. It is therefore important that the inlet and twin 36-inch gravity discharge lines be kept free of sediment of debris buildup.

The interior flood damage potential to be handled by the Paderewski Pumping Station in the event of a major storm runoff without discharge is considered "medium". Because of the flat topography, a rainfall excess in the order of 4 inches would result in 1 to 2 feet of ponding in streets and low areas. However, because the drainage system is combined storm-sanitary, there would be backup of the sanitary system throughout the residential area posing a considerable health, in addition to flood, problem.

The frequency of high river stage requiring pumping at Paderewski is high. The maximum design stage for pump activation is 52 feet NGVD, which occurs on an average of at least once per year. The 60-inch diameter inlet drain to the station has an estimated capacity in the order of 100 to 120 cfs, however, a flow rate of 70 cfs is probably more representative of the overall storm drainage system capacity. It is concluded that the Paderewski pumping station is in need of nearly complete rehabilitation, and the estimated 10 percent annual chance all season runoff rate (in the order of 70 cfs) is considered reasonable design pumping capacity. It is recommended that the station have one smaller variable speed pump plus a minimum of two larger pumps, with physical space for the possible future addition of a third large pump if needed. It is recommended that the existing number and sizing of pumps be retained and that the two larger pumps not be replaced with pumps less than 40 cfs each.

The flood damage potential in the Jones Ferry area, in the event of an intense rainfall without discharge, is considered "medium". Flooding from a 3 to 4 inch excess rainfall would be in the 1 to 2 feet deep in streets and low areas. But it is a combined system, like the Paderewski area, in a residential and industrial area and there could be extensive sanitary sewer backup, posing a significant health as well as flood threat.

The frequency of high river stage requiring pumping is considered high. A river stage of 52 feet NGVD occurs on the average of at least once per year here also. The 10 percent annual chance all season storm runoff is considered a reasonable design at Jones Ferry. This is estimated in the order of 160 cfs, and allowing for 20 cfs from east of the railroad results in a total flow of 180 cfs. It is therefore concluded that the present pumping, with only two of the large pumps and the one small pump operational, is still reasonably adequate. When the present pumps are replaced it is recommended that one small variable speed pump and not less than two large pumps be installed with physical provisions retained for the possible future addition of a third large pump if needed.

The flood damage potential in the Call Street interior area is considered "medium". In the event of 3 to 4 inches of excess runoff without discharge, flooding would be 1 to 3 feet deep in streets and low areas, but because it has a combined storm sanitary drain system there could be extensive sanitary backup. The frequency of high river stage

requiring pumping at Call Street is considered "high". A Connecticut stage of 55 feed NGVD, the maximum pump activation level, has an estimated annual chance of occurrence in the order of 20 percent. The estimated 10 percent annual chance all season runoff rate of 100 cfs (.43 inch R.O./Hr.) is comparable to the present inlet drain capacity, considered a reasonable design capacity under present conditions. Added pumping capacity, unless for reserve backup, would be difficult to justify unless it is determined that the effective drainage is larger and/or improvements were made in the existing storm drain capacity.

Though the combined capacity of one large pump (75 cfs) and a small pump (15 cfs) approach present estimated drain capacity at Call Street, it is recommended that at least two large pumps be maintained operational to provide flexibility of operation and provide a factor of safety against a pump failure. In rehabilitating the station, provision of two large pumps with a lesser capacity of about 60 cfs each may want to be considered.

In Chicopee Falls, the flood potential in the Main Street area in the event of intense excess rainfall is considered "low to medium", provided the upper level pressure drains function as designed. Ponding would be 2 to 3 feet deep tending to concentrate in the low area along the line of protection. The frequency of high river stages requiring pump activation is considered "low". A river stage of 86 feet NGVD, considered the maximum level for pump activation, has an estimated 10 percent chance of annual occurrence. The Main Street station, built in 1964, is in good operating condition. Its design storm runoff capacity of 30 cfs is in excess of the approximate 10 percent annual chance all season runoff.

The flood potential at the Oak Street area in Chicopee Falls is considered "low to medium". One to two feet of ponding would expose some industrial equipment on low level floors, however, potential damages are considered minimal. The frequency of river stage requiring pump activation is considered "low". A river stage of 83 ft NGVD, maximum pump activation level, has about a 10 percent annual chance of occurrence. The station, also built in 1964, is in good operating condition. Design included and additional 20 cfs for industrial process water, which presently is not needed with the industry closed down. The capacity of the station is, therefore, considered highly adequate under present conditions in the area. Two out of the three installed pumps would serve the area quite adequately under present conditions.

VI CONCLUSIONS

A. General

At Paderewski, Jones Ferry and Call Street stations, each equipped with multiple propeller pumps plus a single variable speed volute pump, the design capacity provided by two propeller pumps plus the volute is considered quite adequate for present watershed and storm drain conditions. Rehabilitating the stations to these capacities would be considered a creditable short term goal.

The twin 9 foot square gravity discharge gates at the Dwight Street pumping station were originally sized for the maximum discharge capacity of tailrace number 3- a capacity that has been greatly reduced by filling. Replacing these gates with two smaller 3-foot wide by 4-foot high gates, or the equivalent, should be considered. Also, closing and sealing gates at the no longer used tailrace channels 1 and 2 would eliminate a 15 cfs pumping capacity allowance for leakage, and discounting the 15 cfs originally allowed for industrial process water no longer used would reduce the originally designed requirements to about 80 cfs, this might be provided by two, rather than three pumps.

The Bertha station has no pumping or gravity discharge capacity to spare. Increased watershed area, drainage improvements, and some loss of ponding area makes it necessary that this station be maintained at not less than its full design capacity. The gravity discharge as well as pumping facilities should be properly maintained and clear of any restrictive buildup of sediment or debris at all times.

The Plainfield station was designed for future storm drain improvements that apparently have not taken place. Therefore, the station has more than adequate capacity for present conditions. It is recommended that the station be maintained but that engine replacement be given less priority than at the other five stations. When engines are replaced, possible savings by replacing with less capacity may want to be considered. It is recommended, however, that flood control pumping stations be equipped with not less than two pumps to provide operational flexibility and reduce the risk from a single pump failure. Twin pumps with a total capacity of 10 to 12 cfs would seem appropriate.

B. Need

The six stations at the older Chicopee LPP are in need of systematic rehabilitation including a planned program for pump engine replacement. These stations were originally designed with some allowances for expected future watershed development, but many of the projections have not materialized. At the Dwight station there is a high priority need for the repair or replacement of the twin 9 foot square gravity discharge gates, both presently inoperable.

An increased level of flood protection or extension of the protected areas at the Chicopee and Chicopee Falls LPP's is not needed at this time. The Chicopee LPP provides protection against an event having an annual chance of occurrence approaching 0.1 percent, or one having a recurrence interval of about 1000 years. The Chicopee Falls LPP provides Standard Project Flood protection.

VII RECOMMENDATIONS

A. Modification Advisibility

Modifications to increase the level and extent of flood protection at the Chicopee and Chicopee Falls LPP's are not recommended at this time. However, due to the age of the Chicopee LPP, another review in accordance with EC 11-2-147 should be scheduled. The Chicopee LPP will be 50 years old in 1991. This would be an appropriate time for the next review.

B. Operation and Maintenance

It is imperative that the city of Chicopee continue to recognize its responsibility to properly operate and maintain the projects.

If there is any further delay in repair of the pumping stations by the city of Chicopee, it is recommended Operations Division, NED pursue an immediate solution to bring the conditions back to satisfactory. ER 1130-2-339 outlines the process to be taken. Assistance from the Commonwealth of Massachusetts can be requested. The necessary work could be contracted out by the state and billed to the city of Chicopee or state aid funds held back as payment. The unsatisfactory condition should be made a matter of public knowledge. Use of mass mailing, or the press, are suggested. If the city of Chicopee does not bring the condition back to satisfactory within an appropriate time frame, assumption of maintenance responsibilities by the Commonwealth of Massachusetts is recommended.

VIII CORRESPONDENCE

November 26, 1984

Operations Division, Project Operations Branch

Honorable Richard Lak
Mayor of the City of Chicopee
City Hall, Market Square
Chicopee, Massachusetts 01013

Dear Mayor Lak:

My representatives conducted the semi-annual inspection of the federally built local flood protection project in Chicopee, Massachusetts, on October 30, 1984. I have enclosed a detailed inspection report for your review.

The project is in unsatisfactory condition due to numerous deficiencies listed in the attached report. Call Street, Jones-Ferry, Paderewski Street, Bertha Avenue, and Dwight Street pumping stations all require extensive structural and major mechanical repairs. Numerous water leaks and seepage were evident at all of the above stations. Major cracking of the masonry and large spalls in the concrete were also observed. Many of the pump engines were covered with canopies and plastic sheeting to shield them from water leaks. These conditions greatly increase the risk of failure to the mechanical equipment.

The stone slope protection in the Chicopee section of the project is overgrown with small trees and brush. The growth should be removed and the slopes treated with a state approved herbicide. The growth will be more difficult to remove and the root system may adversely impact the structural integrity of the dike system if left uncorrected.

Funding in the amount of $50,000 has been allocated in the last two years for special maintenance items pertaining to the flood control system. The allocation for the current fiscal year has been reduced to $30,000. Jones-Ferry Pumping Station experienced severe problems during the recent June floods. Only one pump engine was operable and the capacity of that pump was inadequate to discharge the interior drainage which converges on the pump station. As a result, waters were rapidly rising in the ponding area. The city was forced to rent two small pumps and was able to keep the ponding level from rising. In light of the continued need for major replacement and restoration programs which need to be undertaken, I cannot urge you strongly enough to provide the funding for these items. The potential exists for interior flooding due to the inability of the system to pass interior runoff to the Connecticut River.

I would like to meet with you to view the project and discuss your program of remedial actions to restore the flood control system. I have asked James Morocco of my staff to contact your office to set up a convenient time for all parties to meet. I wish to thank your Messrs. Laflamme, Lemay, and Maroney for their cooperation during the inspection. If I can be of further assistance to you, please do not hesitate to contact me at 617-647-8220 or Mr. James Morocco at (617) 647-8291.

Sincerely,

Carl B. Sciple
Colonel, Corps of Engineers
Division Engineer

Copy furnished:

Mr. Frank Rueli
City Engineer
Engineering Department
City Hall Annex
Chicopee, MA 01030

Mr. Ernest Laflamme, III
Flood Control Foreman
69 Silvan Road
Chicopee, MA 01030

Connecticut River Valley
Flood Control Commission
466 Main Street
Greenfield, MA 01301

Basin Manager, LCRB
RCC
PAO
Opers. Div. Files

Mr. John J. Hannon
Chief, Engineer
Div. of Waterways
Dept. of Env. Qual. Engrg.
1 Winter Street
Boston, MA 02108

Honorable Edward P. Boland
Representative in Congress
1550 Federal Building
Springfield, MA 01103

LOCAL FLOOD PROTECTION PROJECT INSPECTION REPORT

Project: Chicopee LPP

Maintaining Agency: City of Chicopee

Type Inspection: X Semi-Annual Staff ____ 90 Day Interim

River Basin: Connecticut Date of Inspection October 30, 1984

Feature	Sat	Unsat	Deficiencies
PUMPING STATIONS - STRUCTURES			
INTERIOR		X	See various Comments
EXTERIOR		X	See various Comments
PUMPS - MOTORS - ENGINES			
TRIAL OPERATED	X		
GENERAL CONDITION		X	See Comments
POWER SOURCE	X		
INSULATION TESTS	N/A		
METAL INTAKES/OUTLETS	X		
GATE VALVES	X		
GATES - DRAINAGE STRUCTURES			
TRIAL OPERATED	X		
GENERAL CONDITION	X		
LUBRICATION	X		
DIKES - DAMS			
GENERAL CONDITION	X		
SLOPES/EROSION		X	See Comments
SAND BOILS/CAVING	X		
TRESPASSING	X		
SLOPE PROTECTION		X	See Comments
DRAINS	X		
STOP-LOGS - LOG BOOM			
CONDITION OF LOGS		X	See Comments
AVAILABILITY OF LOGS		X	See Comments
HIGHWAY SLOTS	X		
STORAGE FACILITIES	X		
CHANNELS - OUTLET WORKS CHANNEL			
BANKS	X		
OBSTRUCTION CONTROL	X		

Feature	Sat	Unsat	Deficiencies
CONCRETE STRUCTURES			
SURFACE	X		
SETTLEMENT	X		
JOINTS	X		
DRAINS	X		
MISCELLANEOUS			
EMERGENCY OPER. PLAN	X		
EMERGENCY EQUIPMENT	X		
SEMI-ANNUAL REPORT			Interim reports due in February and August

Inspection Party:

Ernest LaFlamme - City of Chicopee
Robert LeMay - City od Chicopee
Kevin Maroney - City of Chicopee
James Morocco - Operations Division, NED Corps of Engineers
Joseph Ledgere - Operations Division, NED, Corps of Engineers

Photographs Taken:

None

Remarks & Additional Comments:

(Indicate Here Observations, Discussions, Specific Feature Deficiencies, Recommendations and any other pertinent information. Use Continuation Sheet if necessary.)

See attached Sheets

X ALL APPLICABLE ITEMS. IF UNSAT INDICATE SPECIFIC DEFICIENCIES. INDICATE IF NOT APPLICABLE.

DATE	INSPECTED BY: TYPED NAME & TITLE	SIGNATURE
15 Nov 1984	JAMES A. MOROCCO, Civil Engineer	James A. Morocco

CHICOPEE, MA - LPP INSPECTION

October 30, 1984

The Chicopee Local Protection Project was inspected on October 30, 1984 and the following comments are provided:

CALL STREET PUMPING STATION

a. Buffalo pump engines are obsolete and should be replaced.
b. Engine #1 is inoperable due to blown head gaskets. The replacement parts are not available (see last several reports).
c. Engine #2 operated satisfactorily.
d. Engine #3 operated satisfactorily.
e. Emergency generator was operated satisfactorily.
f. The 16" volute pump was operated satisfactorily.
g. The sump pump is operational.
h. Gravity sluice gate was trial operated.
i. Electric operator for sump intake gate does not work.
j. CO^2 cylinders for fire suppression have been inspected and the lines blown out (also for Jones Ferry)
k. Overhead crane at this station, Jones Ferry, Paderewski, Bertha Avenue and Dwight Street have not had a load test since installation.
l. Pump discharge flap valves require painting (see last several reports).
m. Spalled concrete around pump #1 discharge flap valve still requires repairs.
n. Coping and brickwork need major repairs, there is a large spall and crack at the south corner of the rear wall.
o. Windows have several broken panes of glass and several torn screens.
p. Roof has several leaks which require attention.

JONES FERRY PUMPING STATION

a. Buffalo pump engines are obsolete and should be replaced.
b. Engine #3 is inoperable due to a frozen piston. Replacement parts are not available.
c. Engine #2 was operated satisfactorily.
d. Engine #1 could not be started due to a broken fuel line, fuel tank and line need replacement.
e. Emergency generator was started.
f. Volute pump was operated.
g. Sump pump was operated.
h. Both sluice gates operated satisfactorily.
i. Brickwork requires extensive repairs and the concrete parapet is spalling.
j. Several window block and screens require replacement.
k. Trash racks require cleaning and painting.

PADEREWSKI STREET PUMPING STATION

a. Engine #1 is removed. A replacement engine is on site, but has yet to be installed.

b. Engine #2 operated satisfactorily with use of a battery charger. New batteries are needed. A diesel replacement engine has been ordered.

c. New emergency generator is in place, but has not been wired. Electrician is working and will be finished within 30 days.

d. Volute pump was operational.

e. Sump pump and fan operated satisfactorily.

f. Boiler system is operational.

g. Sump intake and gravity bypass sluice gate are operable. They have been fitted to operate using a portable electric drill operator.

h. Discharge flap valve on pump #1 is cracked and requires repair.

i. Trash racks require cleaning and painting.

j. Building roof requires replacement

k. Interior and exterior of building require extensive repairs. Interior will not be repaired until roof is replaced.

BERTHA AVENUE PUMPING STATION

a. Engine #1 was operated satisfactorily.

b. Engine #2 was operated satisfactorily.

c. Ponding intake area is heavily silted. There has been a considerable build up since last inspection. The area should be periodically cleaned on an as needed basis. An access road is being constructed to facilitate silt removal.

d. Stop logs for railroad closure have been procured.

e. Trash racks have been cleaned, but require painting.

f. Exterior and interior of the station requires waterproofing and painting.

g. Several windows and screens need replacement.

DWIGHT STREET PUMPING STATION

a. Engines #1 and #2 were operated with the use of a battery charger. New batteries are needed.

b. Pump #3 is not connected to the drive engine.

c. Gravity bypass sluice gates are inoperative. Bids for replacement gates are much higher than estimated and monies are not available at this time.

d. Sump intake gates are operable.

e. Sump pump is operational.

f. Exterior brickwork requires extensive repair and waterproofing. There is a major spall and crack on the south wall.

g. Safety treads on exterior concrete stairs require replacement.

h. Exterior concrete deck requires repair.

i. Handrails need painting.

PLAINFIELD STREET PUMPING STATION

a. Engines #1 and #2 operated satisfactorily with the use of a battery charger. New batteries are needed.
b. Insulation is missing from exhaust of engine #1.
c. Bypass sluice gate was satisfactorily operated with the use of a hand crank operator.
d. Pump intake sluice was operated satisfactorily.
e. Embedded steel ladder rungs leading to intake gate are severely corroded and unsafe and should be replaced.
f. Engine radiator exhaust louvers are boarded over. They should be hinged so they may be opened during operations. This also applies to the Main Street and Oak Street pumping stations.
g. Exterior metals (doors, gate enclosures) should be painted.

MAIN STREET PUMPING STATION

a. Engines #1 and #2 were started and ran satisfactorily.
b. Dehumidifier is operational.
c. Sump exhaust fan blade hits housing and housing should be repaired.
d. Sump intake gate does not close. Limit switch should be checked.
e. Sump gravity bypass gate does not close.
f. Interior and exterior of the station is in good condition.

OAK STREET PUMPING STATION

a. Engines #1, #2 and #3 were started and operated satisfactorily.
b. Sump intake sluice gate will only open, close button does not work. This situation should be repaired as soon as possible. (see last report)
c. Two gravity sluice gates were operated.
d. Interior and exterior of the station are in good condition.
e. Wooden stairway leading to dike is rotten and deteriorated in various spots and should be repaired.
f. Rails and walkway at gate structure requires painting. Plastic cover over the gate stem is broken and should be repaired. The walkway, grates and railings of the gate structure at the Oak Street station require painting.

DIKES AND FLOODWALLS

a. Mowing of dikes is required and should be accomplished on a regular basis.
b. Some sections of the stone protection have been cleared. however the slope for the most part are overgrown and the growth should be cut and removed.
c. Floodwall west of the Dwight Street station has a major vertical crack and spalling at the third joint. The floodwall east of the station is in good condition.
d. The floodwall adjacent to and east of the Main Street station is in good condition.

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02254

May 16, 1984

REPLY TO
ATTENTION OF:
Planning Division
Plan Formulation Branch

Honorable Richard Lak
Mayor of the City of Chicopee
City Hall, Market Square
Chicopee, Massachusetts 01013

Dear Mayor Lak:

I have initiated a review of the existing Chicopee local flood protection project, completed by the Corps of Engineers in 1941. This project, like others we are studying in New England, was designed and constructed many years ago using design criteria in effect at that time. Our study will include a review of the adequacy of flood protection currently provided by the project, recent and possible future development in the watershed and new information on basin hydrology. We will also be looking for opportunities to make the project more viable, safe and reliable using current design standards.

Initially the study will be limited to a reconnaissance report which will evaluate the need for any modification to the completed project and determine whether there is a Federal interest in continuing the investigation. If warranted, I may recommend a follow-on feasibility study. During the feasibility study stage any modification plans will be formulated using current design criteria and screened from the standpoints of economics, environmental effects, engineering integrity and safety considerations. Items of local cooperation, both existing and those required for the future, will also be addressed if further action is recommended.

This study is not a substitute for the semi-annual inspections performed by my Operations Division personnel. Those inspections are conducted to ensure that the city is complying with the assurances of local cooperation signed by the city prior to construction of the Chicopee project. This reconnaissance study will utilize previous semi-annual inspection reports and correspondence with the city as background information and will identify existing and potential problems previously observed which should be reviewed as part of this study. A member of my Planning Division staff will participate in the semi-annual inspection of the Chicopee project on May 22, 1984.

Your comments are vital to our study. In the near future, a member of my staff will be contacting you, or a point of contact you appoint, to set up a meeting to discuss our study and hear your viewpoints. If you have any questions or comments, please do not hesitate to call me at (617) 647-8220. Mr. Richard Zingarelli will be managing the study. He may be reached at (617) 647-8557.

Sincerely,

Carl B. Sciple
Colonel, Corps of Engineers
Division Engineer

CORPS OF ENGINEERS

U. S. ARMY

A B C D E F G H

DRAINAGE AREAS

CHICOPEE L.P.P.

- (1) 30 ACRES
- (2) 25 "
- (3) 400 "
- (4) 350 "
- (5) 630 "
- (6) 230 "

CHICOPEE FALLS L.P.P.

- (7) 20.5 ACRES
- (8) 17.8 "

PUMPING STATIONS

CHICOPEE L.P.P.

- [1] PLAINFIELD
- [2] DWIGHT
- [3] BERTHA
- [4] PADEREWSKI
- [5] JONES FERRY
- [6] CALL STREET

CHICOPEE FALLS L.P.P.

- [7] MAIN ST.
- [8] OAK ST.

SCALE IN FEET

0 1000' 2000' 3000'

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

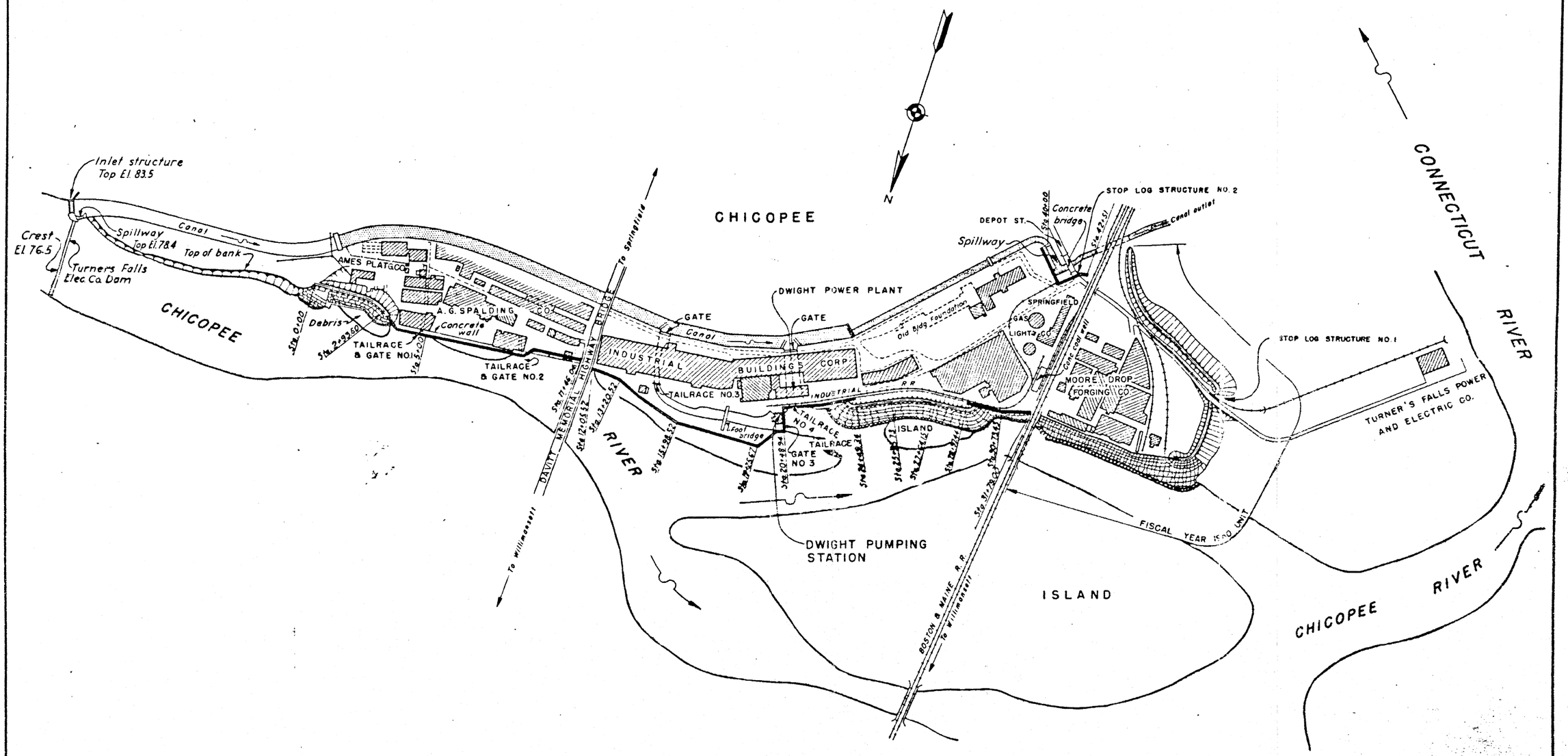
CHICOPEE & CHICOPEE FALLS
LOCAL PROTECTION
CHICOPEE, MASSACHUSETTS

HYDROLOGIC REVIEW OF INTERIOR DRAINAGE

INTERIOR DRAINAGE
AREA MAP

FEBRUARY 1983

PLATE I



CONNECTICUT RIVER FLOOD CONTROL

DIKE-SOUTH BANK CHICOPEE RIVER AND DWIGHT PUMPING STATION

GENERAL PLAN

CONNECTICUT RIVER CHICOPEE, MASS.

SCALE: 1 IN = 200 FT

200' 0 200' 400'

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

OPERATION AND MAINTENANCE MANUAL

CHICOPEE, MASS

PLATE 2

CORPS OF ENGINEERS

U.S. ARMY

PROFILE OF FLOOD WALLS AND DIKES

SCALE: HOR. 1" = 200'
VERT. 1" = 10'

NOTE:
② INDICATES BUILDING NUMBER

GENERAL PLAN
SCALE 1" = 100'

NOTE
See Sheet 3 for location of borrow and spoil areas

Record Drawing
Contract No. DA-19-016-CIV ENG. 64-19

10	6-18-63	Final Field Corrections
9	5-16-63	Spec. No. revised
8	3-14-63	Borrow and spoil area locations, & rail fence rem.
NUMBER	DATE	DESCRIPTION

GREELEY & HANSEN ENGINEERS and CHAS. T. MAIN, INC. BOSTON, MASS. — CONSULTANT-ENGINEERS

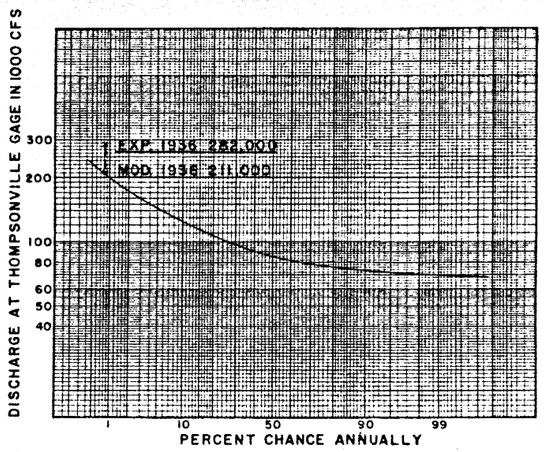
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND, CORPS OF ENGINEERS, WALTHAM, MASS.

CHICOPEE RIVER FLOOD CONTROL
CHICOPEE FALLS
GENERAL PLAN
CHICOPEE RIVER — MASSACHUSETTS

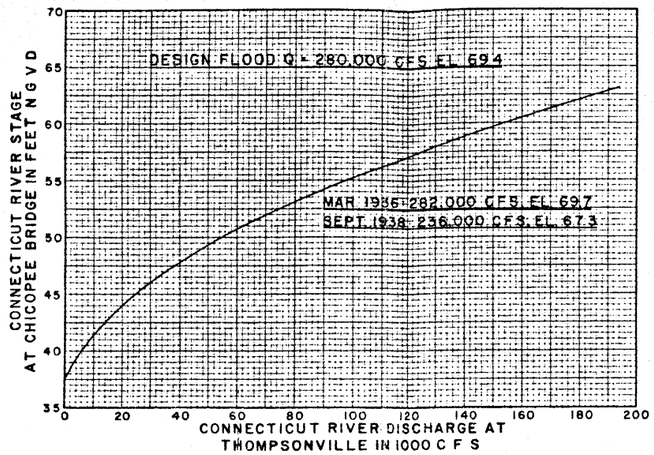
DATE APRIL, 1963

DRAWING NUMBER
CT-5885

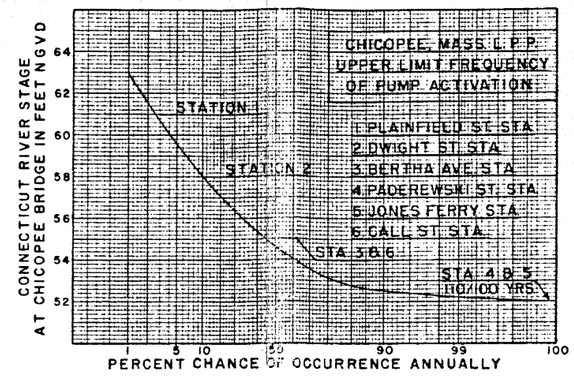
PLATE 3



CONNECTICUT RIVER PEAK DISCHARGE FREQUENCIES
AT THOMPSONVILLE AS MODIFIED BY RESERVOIRS

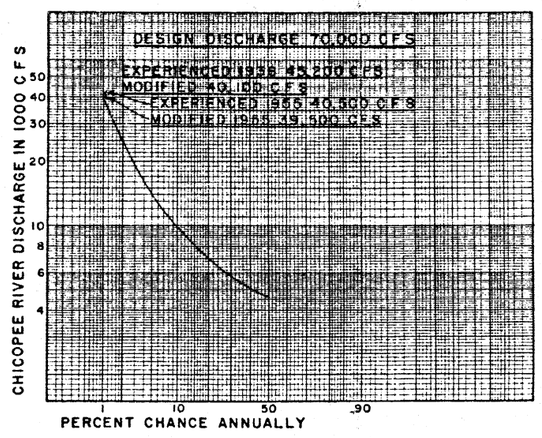


CONNECTICUT RIVER STAGE DISCHARGE RATING

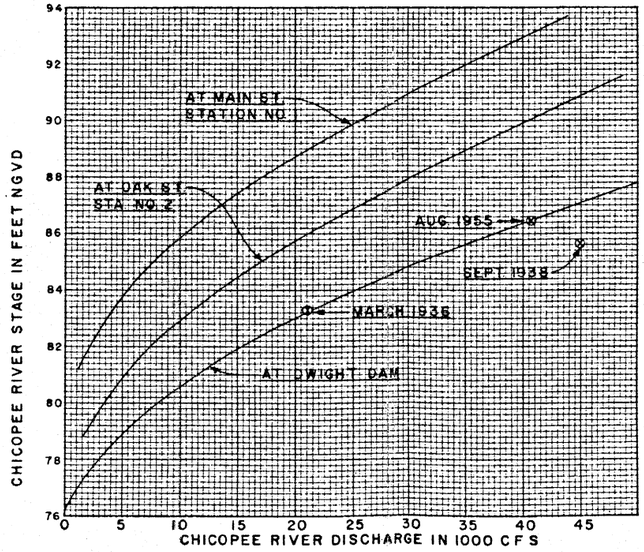


CONNECTICUT RIVER MODIFIED STAGE FREQUENCY
AT CHICOPEE BRIDGE

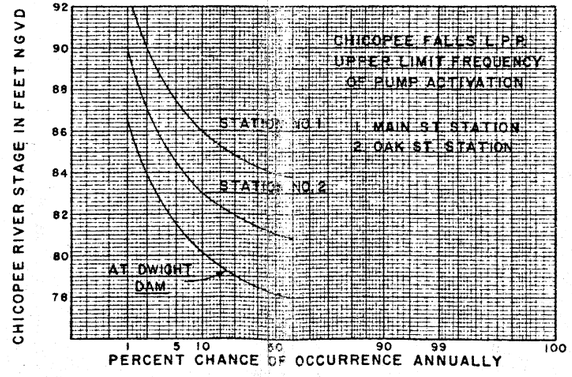
CHICOPEE LOCAL PROTECTION



CHICOPEE RIVER PEAK DISCHARGE FREQUENCIES
AS MODIFIED BY RESERVOIRS



CHICOPEE RIVER STAGE DISCHARGE RATINGS



CHICOPEE RIVER MODIFIED STAGE FREQUENCIES

CHICOPEE FALLS LOCAL PROTECTION

GRAPHIC SCALES

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY | DR. BY | CK. BY

SUBMITTED:
CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, DESIGN BRANCH
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF, PROJECT MGMT. BRANCH

CHICOPEE & CHICOPEE FALLS
LOCAL PROTECTION
HYDROLOGIC REVIEW OF INTERIOR DRAINAGE

RIVER DISCHARGE AND STAGE FREQUENCIES

APPROVED DATE
CHIEF, ENGINEERING DIVISION

SCALE | SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 4

INTERIOR FLOOD POTENTIAL

HIGH: GENERALLY EXCEEDING 5'. EXTENSIVE PROPERTY LOSS. THREAT TO HUMAN LIFE.

MED: GENERALLY 3-5' EXTENSIVE PROPERTY LOSS LITTLE THREAT TO LIFE.

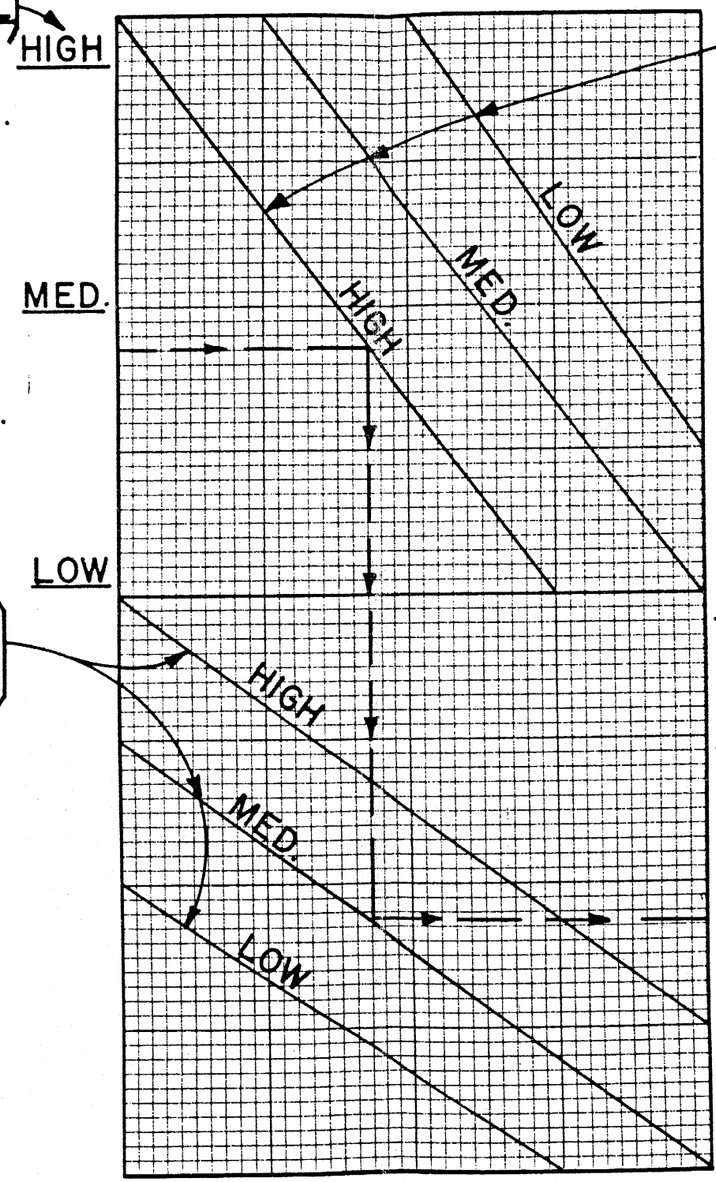
LOW: GENERALLY 1-3' IN STREETS AND LOW AREAS MODERATE PROPERTY LOSS.

INTERIOR RUNOFF AND RIVER STAGE COINCIDENCE

HIGH: INTERIOR RUNOFF AND RIVER GENERALLY PEAK WITHIN 1 TO 4 HOURS OF EACH OTHER.

MED: RUNOFF AND RIVER GENERALLY PEAK WITHIN 4 TO 12 HOURS OF EACH OTHER.

LOW: RIVER RISE GENERALLY MORE THAN 12 HOURS AFTER INTENSE RAINFALL.



FREQUENCY OF RIVER STAGE REQUIRING PUMPING

HIGH: GENERALLY MORE THEN 5 YR. FREQUENCY.

MED: 5 YR. TO 20 YR. FREQUENCY.

LOW: LESS THAN 20 YR. FREQUENCY.

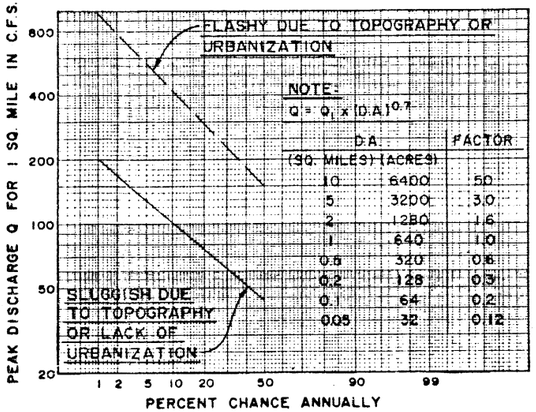
RELATIVE RUNOFF FREQUENCY FOR INTERIOR DRAINAGE DESIGN

CHICOPEE AND CHICOPEE FALLS
LOCAL PROTECTION PROJECTS

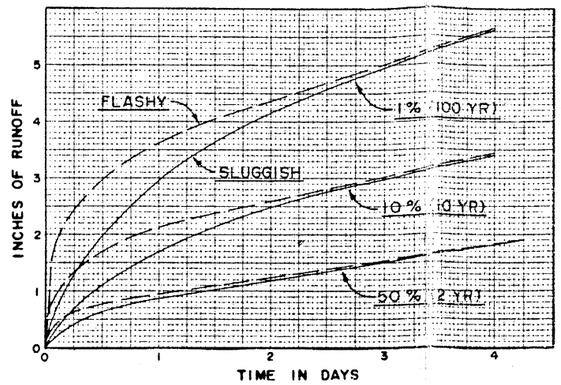
GUIDE CHART FOR HYDROLOGIC REVIEW OF INTERIOR PUMPING CAPACITIES

FEB. 1983

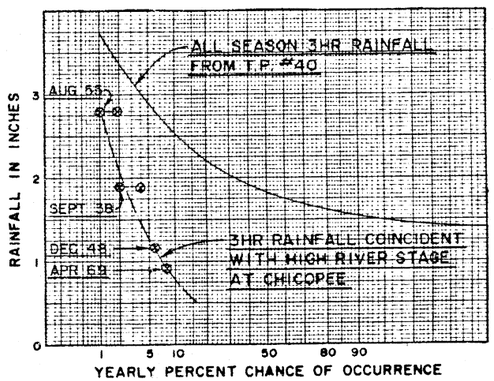
CORPS OF ENGINEERS — U. S. ARMY



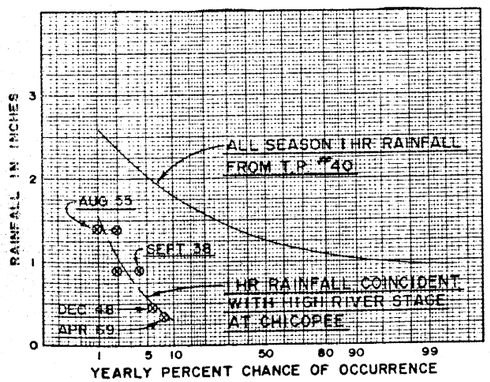
COMPARATIVE PEAK DISCHARGE FREQUENCIES
IN CENTRAL MASS. AND CONN.



STORM RUNOFF VOLUME DURATION FREQUENCIES
IN CENTRAL MASSACHUSETTS



3 HOUR RAINFALL FREQUENCIES
AND COINCIDENCE WITH HIGH
CONNECTICUT RIVER STAGE



1 HOUR RAINFALL FREQUENCIES
AND COINCIDENCE WITH HIGH
CONNECTICUT RIVER STAGE

SAFETY PAYS

GRAPHIC SCALES

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PROJECT MANAGER		
APPROVAL RECOMMENDED:		
CHIEF, PROJECT MNGT. BRANCH		

CHICOPEE & CHICOPEE FALLS
LOCAL PROTECTION
HYDROLOGIC REVIEW OF INTERIOR DRAINAGE
RAINFALL - RUNOFF
RATE AND VOLUME FREQUENCIES

APPROVED _____ DATE _____
CHIEF, ENGINEERING DIVISION

SCALE | SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 6